PLASTIC DESIGN OF MULTISTOREY STEEL FRAMES

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CERTIFICATE

This is to certify that the thesis entitled 'Plastic Design of Multistorey Steel Frames', by Manoharsingh D. Advani is record of work carried out under my supervision and is not submitted elsewhere for a degree.

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ABSTRACT

The present investigation into design aspects of multistorey frames by plastic approach is based on combining modifications suggested by various researchers and incorporating some of own. The effect of adjacent span ratios, spacing of frames, the number of bays, incorporating the frame resistance to horizontal shear, considering the composite beam action, choice of braced bay and higher ratio of column load to yield load have been studied.

To achieve this a computer programme has been developed and the correctness of the same has been checked by comparing the results obtained with previously worked example and the differences have been explained.

For assessing the effects of afore-mentioned modifications two sets of frames have been considered, viz 24 storey frame and 10 storey frame. Considerable economy can be achieved by making proper choice of span ratios etc. The conclusions have been summarised at the end and suggestion for future work have been outlined.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

a	-	Constant;
a_1	-	lever arm of concrete section;
a_2	-	lever arm of steel section;
A	-	area in square feet;
A_b	-	area of bracing member,
A_s	-	area of steel;
b	-	constant;
b_s	-	slab width;
c_1	-	compressive force of concrete slab;
c_2	-	compressive force of steel portion;
C	-	ratio of end moments of column;
C_1	-	load factor for gravity case;
C_2	-	load factor for combined loading;
C_r	-	actual load to yield load ratio;
d	-	depth of column;
d_b	-	depth of beam;
D_L	-	dead load per unit length;
E	-	Youngs modulus;
f	-	shape factor;
F_b	-	bracing force in a storey;
F_c	-	vertical column reaction;
F_{yb}	-	force in bracing member at yield;
h	-	storey height;
H	-	total storey shear;
H_c	-	horizontal column resistance;
H_f	-	frame resistance;
I	-	moment of inertia of section;
I_b	-	moment of inertia of beam;
I_c	-	moment of inertia of column;
K	-	plastic moment ratio of column to beam;
K_b	-	beam stiffness;

K_c - column stiffness;
K_e - effective length factor;
K_n - number of columns at a joint;
K_s - strain hardening factor;
L - centre to centre distance of columns;
L_b - brace length;
L_c - clear span,
L_L - live load
m_o - moment at column end;
M - moment acting at a joint;
M_c - total moment on column;
M_{cv} - moment on column due to shear force;
M_D - fixed end moment due to dead load at joint;
M_{ijb} - moment at (i,j) at bottom of column;
M_{ijt} - moment at (i,j) at top of column;
M_{ijL} - moment at (i,j) position on left face of the column at the joint;
M_{ijR} - moment at (i,j) position on right face of the column at the joint;
M_p - plastic moment of section;
M_{pb} - plastic moment of beam;
M_{pc} - plastic moment of column;
M_u - plastic moment of composite section;
M_y - moment at a distance y from the origin;
P - load on column;
P_{ba} - the axial thrust on beam;
P_{cr} - critical load capacity of the column;
P_y - yield load of section;
r - radius of gyration;
R - percentage reduction;
S - plastic modulus of section;
t - depth of slab in compression;

T	-	tensile force;
V	-	shear force acting on the face of the column;
W	-	weight of section/unit length;
W_D	-	factored uniform dead load/unit length;
W_L	-	factored uniform live load/unit length;
W_s	-	safe load on beam;
W_T	-	factored total uniform load/unit length;
Y	-	distance from origin;
δ	-	elongation in bracing member;
δ_M	-	change in moment at joint;
δP_c	-	additional column load due to wind;
ϕ_{pb}	-	rotation at hinge formation;
Δ	-	deflection of frame and/or column;
η	-	angle of bracing member with horizontal;
σ	-	axial stress in beam;
σ_c	-	cube strength;
σ_e	-	Eulers stress;
σ_y	-	yield stress and
θ	-	rotation of frame or column.

I INTRODUCTION

1.1 General

A structure in its crudest form since times past, to its present sophisticated, aesthetically amazing outline, minimised cum optimised resources inputs and functionally overwhelmingly satisfying refinements represents the constant endeavour of the mankind, its evolution by landmarks can be witnessed either from surviving ageless masterpieces, or, can be traced by archaeologists through excavations. A single most important aspect of man's effort to tame environment and conquer nature has its foundation in a structure. In one or the other of its countless forms, be it a packing material, skyscraper, a mighty rocket hurled into sky with tremendous thrust or a bulk carrier subjected to unceasing wave action, it is the structure which protects the occupants within and defies the environment outside. In all these cases, the structure has to be designed to perform under different types of conditions with a highest common factor being the satisfactory containment/transference of forces. Each structure has to be, therefore, tailor made to perform with utmost reliability, commensurate with over-all economy.

For providing accomodation to exploding population primarily as residential quarters and business activity centres which is further accentuated by heavy concentration of resources in metropolitan and highly urbanised centres, resulting in paucity of us-able land, it was only natural for man to utilise the space in third dimension which resulted in early twentieth century skyscrapers, the outstanding example of first generation being the Empire State Building (1250 ft.).

However, as problems mentioned heretofore multiplied, one more factor, viz., rapid escalation of construction cost, which had revolutionising effects on design philosophy of these types of structures added a new dimension to the now completely defunct approach to design.

1.2 Statement of the Problem

With continuous increase in pressure on urban lands and sharp increase in construction cost, the present trend is to design multistorey frames of a structure to carry vertical loads, resist horizontal shear and to minimise the use of components which performed secondary role. With ever greater understanding of response of material under stress conditions and the ability to produce a construction material with highly consistent engineering properties conforming to idealised behaviour has led to utilise, hitherto backup, strength in nonlinear response zone of the materials. This has an advantage in obtaining uniform load factors for structural elements.

The structures may be classified into various groups depending on geometry, dimensional ratio of sides of elements and purpose. The framed structure may be classified as:

a. plane frames; braced or unbraced and

b. space frames.

In general higher the indeterminacy of a structure greater is its reserve strength in nonlinear response zone. It is an accepted practice

to determine working strength within the elastic zone. In frames, the indeterminacy is usually high, because of the assumption that the joints and the connections are infinitely rigid.

To reduce an indeterminate structure to a determinate one, through failure process, the approach is to load it and let the stress concentration occur at predetermined points. There is then a limit reached beyond which the stress concentration cannot increase, although the load may increase on the structure. At this stage, when the element from top to bottom fibre is stressed to yield limit, the structure behaves in that region as hinged, with a constant maximum moment and the process of internal redistribution then commences in general. As these points increase in number, the degree of indeterminacy is reduced and at one stage the structure becomes determinate. Beyond this point, when structure is loaded a mechanism is obtained finally resulting in failure. It is using this plastic approach, that the frames are analysed and cost effectiveness achieved. However, formation of a mechanism only does not lead to an optimised solution. As will be subsequently seen, the optimisation is achieved when all possible mechanisms occur simultaneously. In other words, all the sections, when stress concentration occurs, should yield at the same time to obtain a complete collapse. This is still in substructure optimisation stage, considering the structure comprising several frames, which may have any number of storeys and bays, there may or may not be any constraints on spacing and sizes of frames for a given enclosed area, and the foundation of frames. In entirety, therefore, minimisation will be a difficult proposition, if not, an impossible aim to achieve.

1.3 Forces

A high rise structure has to withstand gravity loads, wind loads, seismic loads and temperature stresses. The first of these can be classified as manmade and the remaining as natural due to geophysical processes.

The forces can be further classified as deterministic and probabilistic. In probabilistic approach the seismic and wind excitation analysis in multiple load combination deals with vibration of structure and the aim of analysis is to obtain the mode shape there by obtaining the maximum displacement and stresses within predicted life span of the structure. At times this approach may lead to cost excalation. The various factors required for this type of analysis are laid down in relevant codes of practice.

1.4 Structural Engineering and Interaction

A structural engineer has to act in concert with the owner, to satisfy him about the safety of the structure and its efficient end use at the most economical cost, which will include initial construction costs, repairs and maintenance over predicted life span. He has to take into account architectural aspects of structure and has to provide enough servicing space for maintenance of environment as envisaged by a code and as dictated by the projected use of the space created. There may be further design constraints resulting from the limited range of the material availability, the erection process, the construction space, the time limits etc.

All these factors have profound effect on the manoeuverability of the structural engineer and on the design philosophy, which aim at obtaining efficient end products.

1.5 Multistorey Frame Design

The high rise structure may have the skeleton system designed as a truss or a frame. In practice rectangular frames are widely used. The frames may be provided with bracing to resist horizontal forces. The bracing is a comparatively economical alternative to moment resisting frame.

1.6 Objective and Scope of Investigation

The objective of present study is to find a blended mix of presently advocated philosophies to obtain an efficient planar framing system.

To achieve this, the following system has been adopted:

i) A computerised programme based on the Lehigh (1965) design philosophy is developed.

ii) Various modifications are incorporated and efficiency of different approaches compared.

The study has been confined to multistorey, multibay, uniform, rectangular planar frames subjected to deterministic wind and gravity loads, with a multiple loading combination.

1.7 Design Philosophy

The approach to design of the multistorey multibay frame is to discretise the structure and design each storey separately, from top to bottom. To obtain a first approximation of the member sizes the gravity loading has been considered. Subsequently each element in conjunction with a limited number of neighbouring elements has been used for its ability to withstand the multiple combination of the loading. The design of an individual member in this manner is termed as subassemblage approach. The subassemblage has been subsequently dealt with in relevant chapters.

The design of bracing is based on an assumption that the braced bay acts as truss and horizontal force is resisted by trussed bay beam, column and bracing member. The section of the beam and the column in a braced frame may have to be revised. In Lehigh approach the frame resistance has been neglected, while in the present case the saving in weight due to consideration of frame resistance has been worked out. The bracing may also be staggered to avoid revision of column section due to over loading on column.

Although it will be advantageous to let all the four hinges develop in a sway mechanism, to restrict the horizontal deformations only three hinge formation is acceptable.

1.8 Literature Review

The plastic method of designing a structure became essential to minimise the weight and, therefore, developed vigourously as aerospace structures became more common. However, for civil engineering structure plastic design concepts were recommended only in 1958 by American Institution of Steel Construction for frames upto two storeys. At present, however, multistorey frames of any height are being designed by plastic design approach. The outstanding example being 87 storey Standard Oil of Indiana Building.

Wright and Gaylord (1968) evolved the method of incorporating the nonlinear effects of frame deflection and considered equilibrium in the deformed state. They used iterative technique and at each repetition, the equilibrium condition with previous deformation has been checked. The approach is unique, in that, when the tangent to load-deformation curve becomes almost flat, the evaluation is still possible. This has been achieved by using fictitious spring with linear characteristics. This method however has been suggested for refined calculations.

Leroy and William (1970) have suggested a method of determining the cost gradient for many alternatives that are available, viz, braced and unbraced bay and for these conditions the evaluation for each bay has been made. The problem, thus, has been reduced to gradient search technique. The authors have concluded that the structure with minimum weight may not result in practical layout of bracing.

Leroy and William (1972) have used storeywise approach of finding the minimum cost for a given frame. They apparently felt that regorous mathematical optimization techniques may not always be suitable for design office use and have used heuristic method discussed in their previous paper. In this approach, though, the axial load increment on column, has been considered while developing the sensitivity coefficients, this does not appear to be an exact solution, because if the trussed bay position in lower storeys changes, the validity of sensitivity coefficient is lost. The authors do not make the claim for the best optimum solution.

Wright (1972) discussed the approximate method of finding the frame resistance. The approach, as discussed in Lehigh (1965), while calculating the storey shear capacity, the resistance of the frame has not been considered. In general there will be considerable frame resistance available due to change in the load factor when the frame is designed for different loading combinations. This frame resistance can be calculated and if sufficient to stand storeyshear bracing avoided.

Heiderbrecht and Smith (1973) have developed a mathematical model for high rise structures. The method has hand application capability for uniform and nonuniform structures for static analysis and for uniform structures for dynamic analysis. The design curves have been developed for ready use.

Fielding and Chen (1973) have discussed the effects of considering the size and stiffness variation caused by the connection. Using slope-

deflection equations and suitably modifying these, due to the effects of connection, an additional equation per joint has been obtained for the additional degree of freedom. They have concluded that ultimate load cannot be improved upon due to additional connection stiffness but drift is reduced at working loads.

Picardi (1973) has discussed a high rise structure. The basic postulation made is that the tower behaves like a cantilever beam. Further it has been shown by comparison, that economy can be achieved by this approach, compared to the high rise structures previously designed and executed, eg Chase (NEWYORK) has steel content of 55 lbs/sft for 60 storey height, as compared to this, the Standard Oil of Indiana Building has 33 lbs/sft steel content for 87 storey height. The outer shell of building acts as a monolithic tube in which columns are rigidly connected with steel plates, thus providing shear continuity essential for beam approach. The inner perimeter of columns have deflection properties independent of outer perimeter. Other connected aspects like fire proofing and foundations have also been discussed.

1.9 Critical Review

With better understanding of behaviour of steel in particular and other construction materials in general, it has been made possible to shrink the requirement of steel and obtain lighter structures. However, the storeywise minimisation as discussed by Leroy and William (1972)

does not always give a practical bracing solution. The latest trend to design the tower as cantilever beam, as in case of Standard Oil of Indiana Building, requires faster and bigger computers and further minimisation may be possible if all shells are interconnected, which will require even more computer effort and machine time.

II CONCEPTS AND ASSUMPTIONS

2.1 General

Steel or reinforced concrete may be used as basic construction material for multistorey frame. Steel is preferred for various reasons, primarily because of ease in erection, comparative lower self weight and better response to excited vibrations due to wind and earthquake forces. The principal property of steel to undergo large deformations at almost constant resistence offered by it, viz, ductile behaviour, has been made use of to develop plastic theory.

Foulkes (1953) was perhaps the first to have advanced theory for a simple frame. Kist (1917) intuitively suggested the static theorem for which the mathematical proof was given by Gvozdev (1936). Prager (1959) gave proof for kinematic theorem which makes available a set of loads as upper bound to safe set of loads. Horne (1950) discussed the uniqueness theorem which gave an exact safe load for a structure.

Foulkes mechanism approach apparently becomes unwieldy for high rise structures with multiple bays and has not been favoured by investigators.

Although the uniqueness theorem gives the exact collapse load, the problem for designer is to select suitable section for a specified load factor and this approach, as discussed by Foulkes (1959) and further explained by Neal (1963) through four theorems and applied to two structures arranged as single storey in case I and as two storeys in case II, appears

to be rather inefficient for that size of problem. In practice multiple loading combinations have to be considered. The fundamental assumption as given by equation

$$W = a + b\,M_p \tag{2.1}$$

where

W - weight/unit length,

a, b - constants and

M_p - the plastic moment of section

holds only for small range of variation in M_p as discussed by Neal. Further assumption is made, that the range of available sections is infinite, which is not true in practice.

Boulton (1952) suggested minimisation of weight of multistorey structure by considering each storey separately. It is this approach which has been favoured by some investigators and has been adopted in the present work. However, no claim for mathematical optimisation is being made. The members have been so chosen by heuristic optimisation, in that these are just sufficient for one of the loading cases and are over-safe for the remaining cases of loading. The optimisation approach as professed by afore mentioned researchers have in general two drawbacks, as far as, practical application is concerned. Firstly, the frames are unbraced, therefore, per necessity, have to develop large bending moments. This is

not the best way of using the material. Secondly, for achieving maximum economy, as defined by mathematical optimisation, unless additional constraints are imposed, all the elements in frame should reach their maximum capacities simultaneously. In case of frames the hinges should form at susceptible stress concentration points at the same time so as to achieve complete collapse. If in practice, this be envisaged, the deformations become enormous and are unacceptable by codes of practice in particular and from occupation comfort point of view in general. Therefore, in its strictest sense, it may never be possible to design multistorey frames based on the foregoing philosophy. Considering all the points in the foregoing discussion, viz, limited number of available section sizes, large variation in plastic moment of elements used in structure satisfying various loading conditions, restricting the deformations to practical limits, even if other practical and related aspects such as fabrication costs, the connections resulting from changes at each storey level of frame ie. obtaining nonrational application and the foundations, which may have considerable effect on overall economy of frame, are not considered. Leroy and William (1972) even after having adopted storeywise approach and having formulated cost gradients for different alternatives available, conclude that the absolute minimum cost configuration obtained may not be practically suited.

2.2 Features of Structure Considered

2.2.1 The n-storey and m-bay structure shown in Fig. 2.1 has the following characteristics.

a. The structure is regular and rectangular in shape. The storey heights are equal. It has been found economical and practical to restrict the slenderness ratio of columns to less than 40. It is, however, not essential to have all storeys of the same height.

b. It is assumed that in transverse direction the structure is comparatively long and all the gravity load is distributed as uniform load on beams and wind load as point load at beam column junctions.

c. Axial deformations and consequent deflections have been neglected.

d. Joints are immensely rigid and elements framing into it maintain constant circular relationship till collapse load.

e. Out of plane bending has not been considered, the structure being sufficiently braced.

2.2.2 Behaviour of Structure: The structures which have atleast one redundancy can be designed by plastic design method. In frames the redundancies are normally high. After first plastic hinge has formed, the

process continues and next highest concentration point generates another hinge. This is called redistribution of stress. The load deformation history to collapse of a redundant structure, in general, has characteristics enumerated as follows:

i) The load-displacement curve shows an elastic zone.

ii) The curve may show the zone of contained plastic flow. During this stage, the curve may undergo more than one change of slope depending upon loading and section properties as shown in Fig. 2.2(a). However, there may be sudden collapse, even if the structure has redundancies as shown in Fig. 2.2(b). This again is a function of relative section properties and loading. In latter case all hinges form simultaneously, therefore, there is no zone of contained plastic flow and hence no stress redistribution occurs.

2.3 Assumptions

The assumptions made in plastic theory and design of structure are enumerated in paras 2.3.1 and 2.3.2 respectively.

2.3.1 Assumptions in Plastic Theory:

i) The material is isotropic and homogeneous.

ii) In beam element for tensile zone the properties are similar to that in direct tension.

iii) The load-deformation relation is, as shown in Fig. 2.3, idealised elastic plastic.

iv) The equilibrium in undeformed state is considered unless stated otherwise.

v) The loading is proportional.

vi) The failure does not take place due to buckling, i.e., the structure is stable till collapse load is reached.

vii) The deformations about the major axis of section, only are considered.

viii) The torsional buckling, local buckling of flange and web, shear and axial force effects and rolling stresses are not considered unless mentioned.

ix) The beneficial strain hardening effects are neglected.

x) The connections transmit the generated plastic moments without deformation. This results in plastic continuity.

2.3.2 Assumptions in Analysis:

i) In case of beam mechanism the hinges form at the faces of column.

ii) When the rotation is 0.004 radians, the bracing yields.

iii) The braced bay behaves as a truss.

iv) The frame resistance is neglected for evaluating bracing force.

v) When wind acts on a frame the windward end hinge disappears in the beam.

vi) The load-deformation characteristics under working load conditions are in the elastic zone.

2.4 Scope of Study

The design cases considered are

a. the gravity load case at load factor 1.7 and

b. the gravity and wind load case at the load factor 1.3.

2.4.1 Storeywise Analysis: The structure is designed as one storey frame from top to bottom, assuming that the storeys above and below the storey under design are safe. However, by such assumptions the continuity is destroyed, which is again incorporated by considering the subassemblage which embraces two storeys. The subassemblage approach is utilised to discretise the structure into a more comprehensible group of elements which can be handled with ease and in doing so some approximations have been made which are dealt with in the relevant chapter.

2.4.2 Subassemblage: The subassemblage discussed in the foregoing article may be defined as a substructure comprising beam and column elements, all in continuous neighbouthood of the joint. The subassemblage so considered is without prejudice to the overall design/analysis of structure and

satisfying the equilibrium and compatibility conditions which can be incorporated in parent structure without resulting in any modification to the adjacent elements of the subassemblage. The subassemblage approach makes the entire structure amenable to simplified, comprehensible, safe and nearest to the optimal design as illustrated in subsequent treatment. The idealisation of subassemblage elements is as shown in Fig. 2.4. It may be noted that idealisation of subassemblage will be governed by adjacent span ratios, loadings and positions of columns.

2.4.4 Moment-Curvature Relationship (M-Φ Curves)

a. For various loading configurations of a beam column, it is essential to determine the load capacity of the column. To avoid lengthy calculations M- Φ curves have been obtained for ready use. The moment-curvature relationship is a function of column load-yield load ratio, slenderness ratio, the ratio of end moments acting on column. The Fig. 2.5 shows a column and its deflected shape and Fig. 2.6 shows a typical M- Φ curve.

b. Column deflection curve (CDC): The column deflection curve by definition is half symmetric wave-length which gives deflection of a hypothetical column in equilibrium under axial load, within this length can be incorporated the original column, with actual loading, which includes axial force and end moments. It may be noticed if end moments are equal in magnitude and have same

sense or opposite in sense, then the hypothetical deflection curve coincide with actual deflection curve as illustrated in Fig. 2.5. The general equation for deflection of the column is

$$- EI Y'' = M_y \tag{2.2}$$

where

E - Youngs modulus of the material,

I - moment of inertia of the section,

Y - distance from origin, and

M_y - moment at distance Y from origin.

The agreement between CDC and actual column deflected shape is not exact, except for $C = -1, +1$ or $m_o = 0$, where C is end moment ratio and m_o is the end moment. However, if deflections are small, the CDC is very close to the actual deflection curve. The CDC has been obtained by integration over increased length, increase in the length being such that the hypothetical column obtained is only axially loaded.

'' Prime indicates differentiation with respect to length along the axis of element.

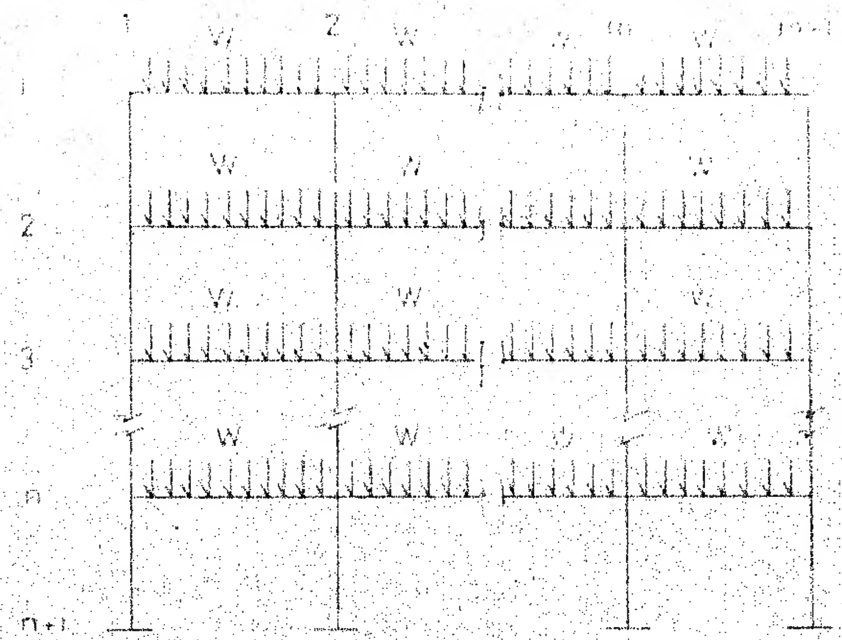


FIGURE 2.1 LAYOUT OF A FRAME

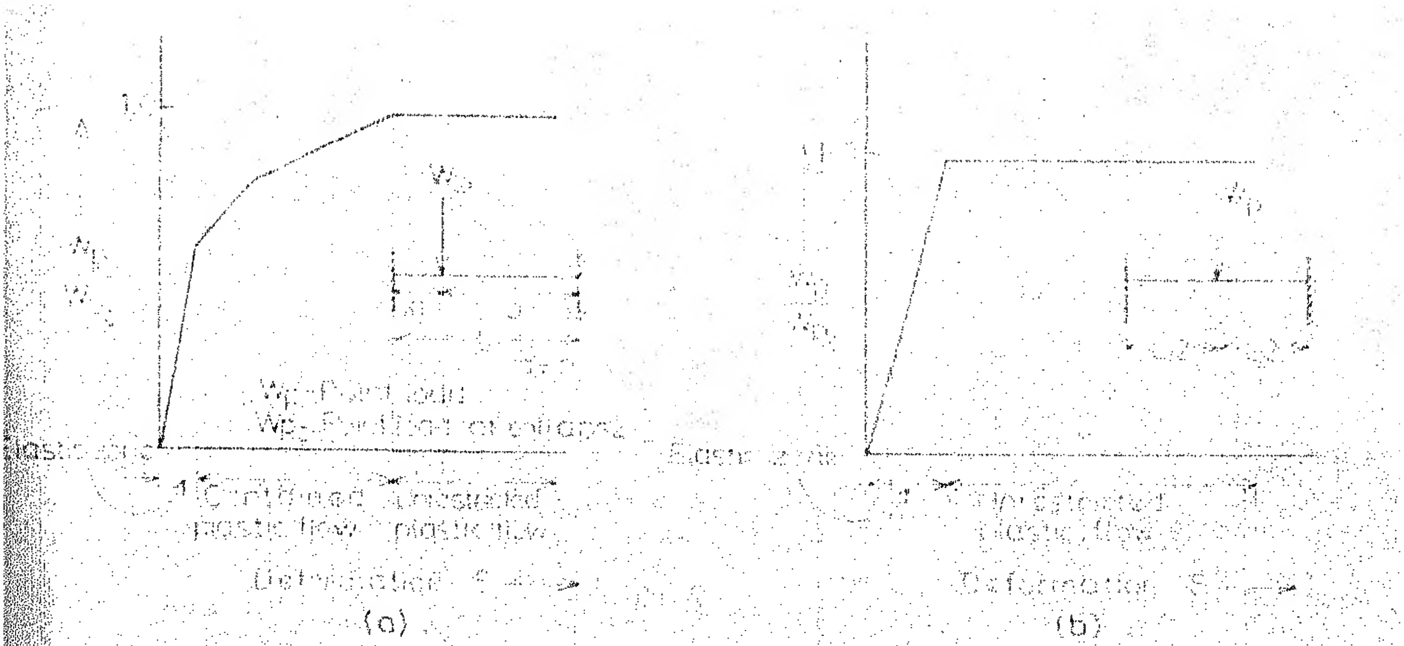


FIGURE 2.2 LOAD-DISPLACEMENT RELATIONSHIP

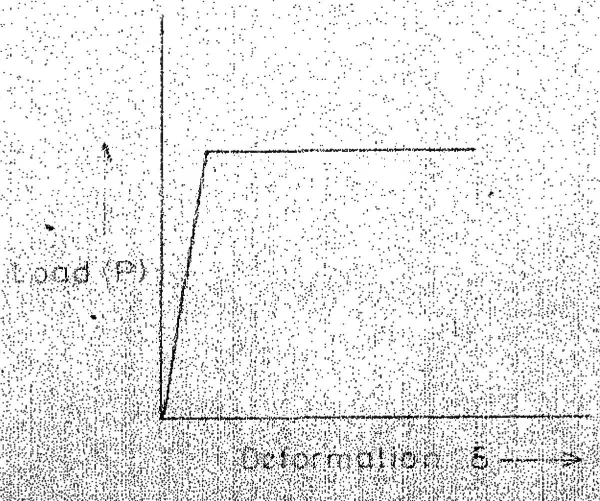


FIGURE 2.3 IDEALISED LOAD DISPLACEMENT

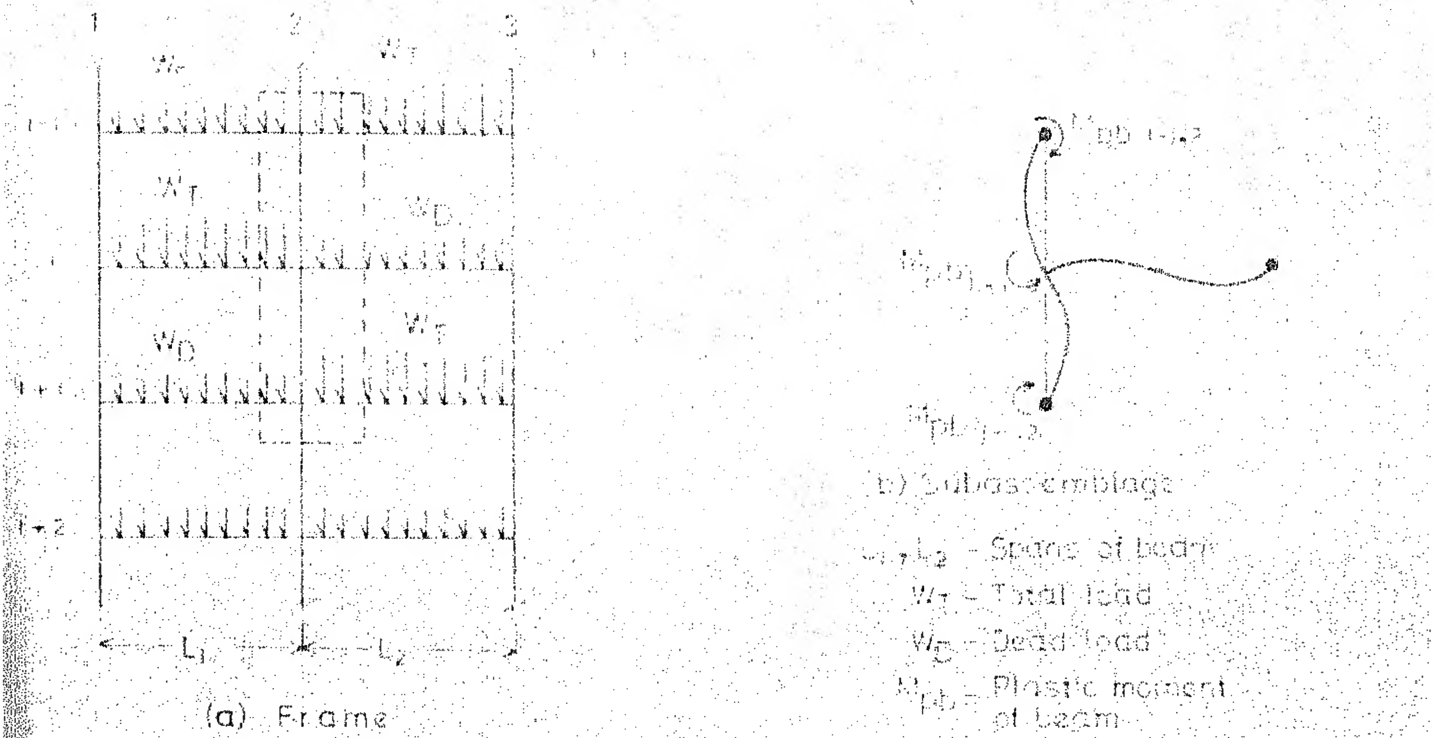


FIGURE 2.4 IDEALISED SUBASSEMBLAGE

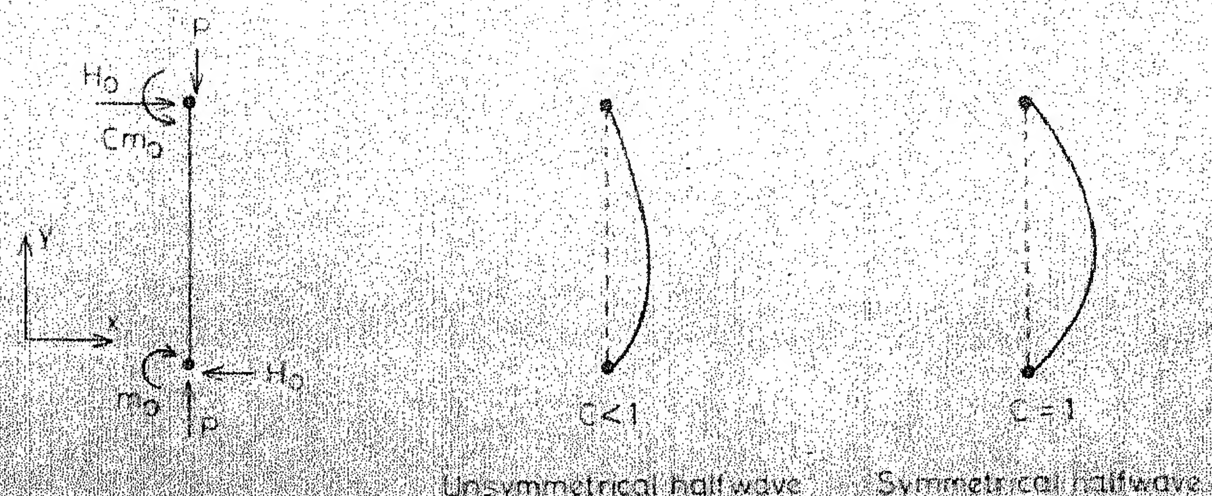


FIGURE 2.5 LOADED COLUMN AND COLUMN DEFLECTION CURVE

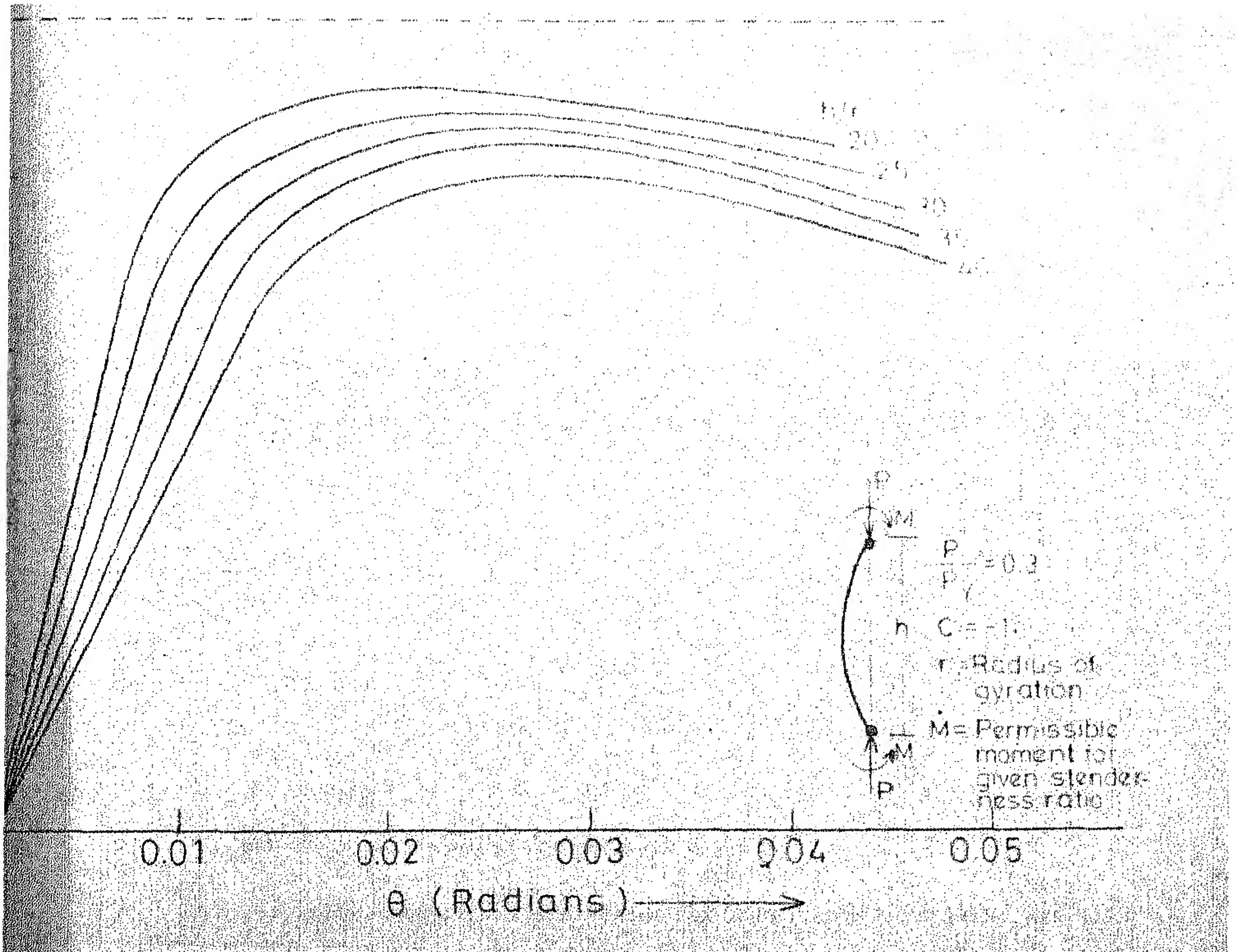


FIGURE 2.6 MOMENT-CURVATURE RELATIONSHIP

III PRELIMINARY DESIGN

3.1 Beam

It is a structural member which supports transverse load and may have a very small axial force, in proportion to transverse load, if present. The beams here in considered deform about major axis.

3.1.1 Experimental Observations: The following experimental observations mark deviation from the assumptions already made.

a. Yielding is not concentrated at a point as discussed by Beedle (1958)

b. Due to presence of residual stresses, yielding commences earlier than σ_y is reached

where $\sigma_y = M_p/S$ . . (3.1)

and M_p - plastic moment of section;

S - plastic modulus of section;

σ_y - yield stress.

c. The strain hardening is present and may increase the capacity of the beam as discussed by Massonet and Save (1965).

d. The maximum moment is maintained for considerable rotation but not for indefinite amount of deformation.

3.1.2 Functions of a Beam: The various functions performed by the beam are,

a. to support the transverse loads,

b. to assist frame in supporting the horizontal loads and

c. to provide lateral stiffness to the frame.

3.1.3 Method of Analysis: A beam under a given set of loads may form one of the following mechanisms:

a. Beam mechanism.

b. Subassemblage mechanism.

c. Sway failure mechanism.

d. Combined failure mechanism.

The analysis may also be governed, apart from the mechanism, by stability criterion as well.

3.1.4 Moment-Curvature (M-ϕ) Relationship: The prediction of beam behaviour in flexure is based on M-ϕ relationship shown in the Fig. 3.1. The assumptions made are,

a. the plane sections remain plane,

b. the crosssection retains its shape throughout the deformations of interest and

c. the beam has no deformation of consequence out of plane of flexure.

In practice these assumptions are not valid in elastic zone. Yet the comparison is close between theoretical and experimental results as is reflected by Fig. 3.1.

3.1.5 Residual Stresses: It has been observed from experimental data that difference between theoretical and actual moment-curvature relationship as reflected in Fig. 3.2 and for various sections is small. The effect as

shown is in the region when yielding first commences. However, irrespective of magnitude of the residual stresses the value of plastic moment remains unattered..

3.1.6 Strain Hardening: The plastic analysis is based on the idealised M-ϕ relationship. To simplify the design approach the strain hardening effects are neglected. In actual behaviour its effects are as follows:

a. Actual load capacity is larger than the theoretical computations as shown in Fig. 3.3.

b. The actual moments at the collapse load for strain hardened material are not equal to plastic moment at hinge locations but tend to be higher at supports by 15%, approximately, and are depressed at centre by the same amount.

3.1.7 Buckling and Bracing of Sections: These are not considered and it is assumed that the width to thickness ratio of compression flange and depth to thickness ratio of web are within permissible limits and beams are adequately braced.

3.1.8 Preliminary Selection of Beam Sections: The beam has to carry dead an live loads. Initially the beam is designed for gravity loading case and the section so obtained is subsequently checked for safety against wind thrust.

3.1.9 <u>Reduction in Load</u>: Only the live load is reduced by minimum of the three percentages which are given as follows:

a) $R = 0.08 \times A$ . . (3.2)

b) $R = \dfrac{100\ (DL + LL)}{4.33 * LL}$ . . (3.3)

c) $R = 60\%$ . . (3.4)

where A - area in sqft supported by member,

R - reduction in percentage,

DL - dead load/sqft and

LL - Live load/sqft.

Reduction is not to be applied for roof beams.

3.1.10 <u>Loading Case</u>: For beams with gravity loads

$$M_{pb} = \frac{1}{16} * W_T * L_c^2 \qquad . . \quad (3.5)$$

where M_{pb} - plastic moment of beam,

W_T - factored total uniform load/unit length and

L_c - clear span.

At corner joint of beam and column, the hinge may form in column as shown in Fig. 3.4. If for calculation purpose the hinge formation is assumed for both ends at face of the column but plastic moment capacity

used is that of the column, no appreciable error will be introduced. The plastic moment is given by

$$M_{pb} = \frac{1}{8} * \frac{W_T\,(2L_o + d)(L_o - d)^2}{(3L_o - d) + K(L_o - d)} \qquad \ldots (3.6)$$

where d - depth of column and

K - ratio between column plastic moment to M_{pb}.

3.2 Column

3.2.1 Loads: It is a practice to assign a column, loads in proportion to the tributary area. This may not be a very accurate assumption in case of elastic analysis where its origin lies, however, it may be very insignificantly inaccurate in case of plastic design approach where system reduces to determinate structure at failure.

3.2.2 Reduction in Live Loads: The live loads are reduced in similar manner as in case of beams, except that area in equation (3.2) is cumulative and, therefore, after few storeys from top the reduction is governed by remaining two equations (3.3) and (3.4). There is no reduction permitted for column supporting roof.

3.2.3 Moments acting on Columns: Clear span as considered for beams, results in additional moments due to shear force in beams and is as given by

$$M_c = \frac{W_T}{16} * (L^2 + 2L * d - 3d^2) \quad \ldots (3.7)$$

where M_c - moment on column due to shear force and load W_T and

L - centre to centre span of beam.

3.2.4 Total Moment on a Column: The moments acting at column ends are algebraic sum of moments at both faces of column. In case of first and last columns in a storey the moment will be acting only on one face of the column. This moment may be shared by two columns depending on the position of the column. The column moments (refer Fig. 2.1) for position (i,j) in the frame are given by

$$M_{c\ i,j} = M_{ij}/K_n \quad \ldots (3.8)$$

where $i = 1, n,$

$j = 1, m + 1,$

M_c = total moment on column

M_{ij} = absolute value of algebraic sum of moments at joint (i,j).

The equation (3.8) can be written as

$$M_{i,j} = \text{ABS} \left[(M_{pbi,j-1} - M_{pbij}) + (M_{cv\ i,j-1} - M_{cv\ i,j}) \right] \quad \ldots (3.9)$$

for $\quad i = 1,\ j = 2,m,\ K_n = 1;$

for $\quad i = 2,n,\ j = 2,m,\ K_n = 2;$

and for $\quad i = 1,n,\ j = 1$ or $m + 1,\ K_n = 1$ (only one beam to be considered)

where $\quad K_n$ - number of columns at the joint

3.2.5 Preliminary Column Design: As an initial trial, a suitable actual l load to yield load ratio C_r is assumed

$$C_r = P_{i,j} / P_{yi,j} \qquad \ldots (3.10)$$

where $\quad P_{i,j}$ - load on column at i,j position and

$P_{yi,j}$- yield load of trial section assumed.

The column so obtained is checked for safety against combined axial load and moment at the ends. The ratio C_r is low for upper columns and high for lower ones. The above calculations are made assuming that the column length is zero. Subsequently the column section so obtained will be checked for actual slenderness ratio by subassemblage method.

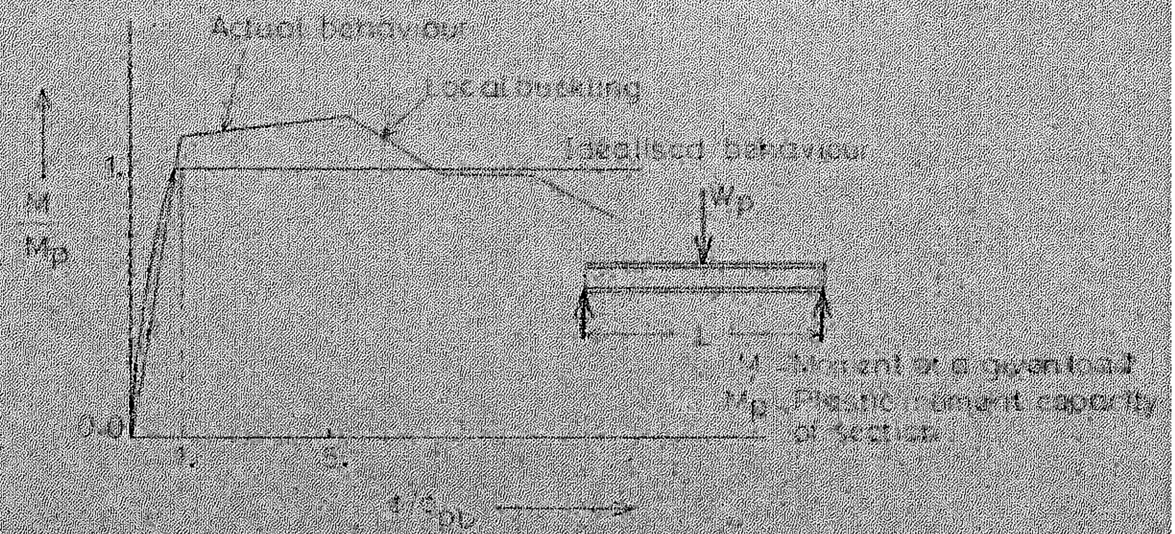


FIGURE 3.1 MOMENT ROTATION CURVE

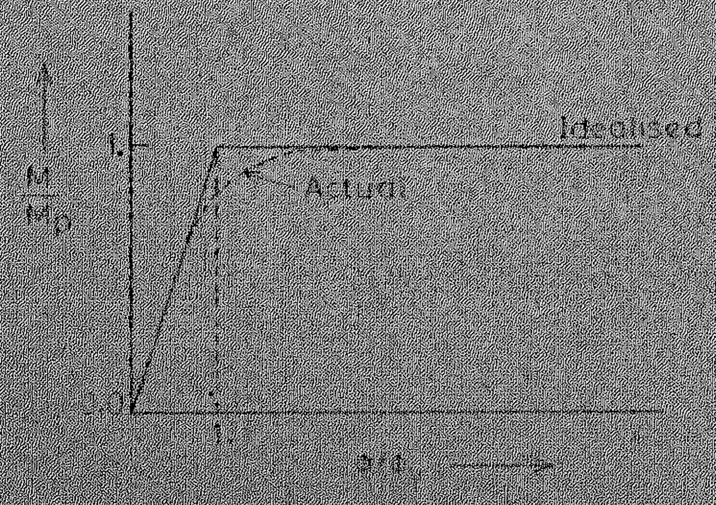


FIGURE 3.2 RESIDUAL STRESSES EFFECT

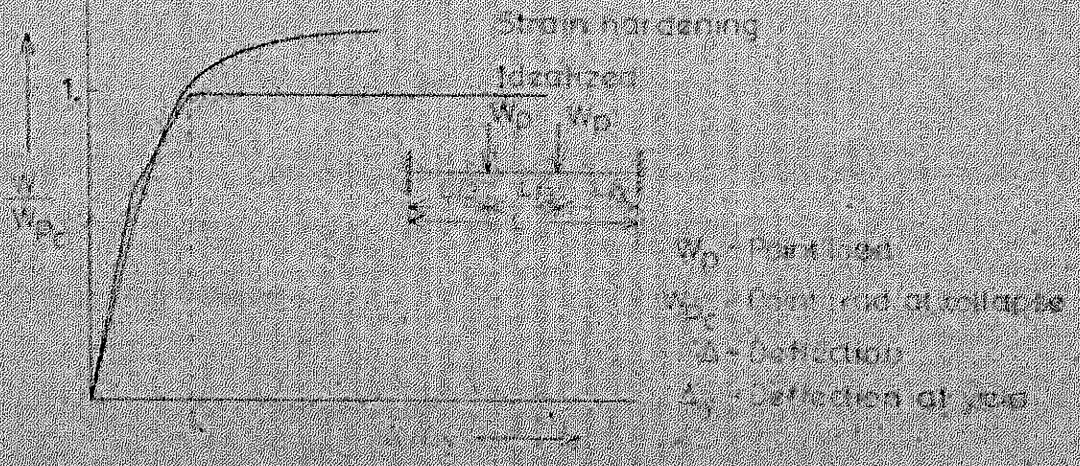


FIGURE 3.3 STRAIN HARDENING EFFECTS

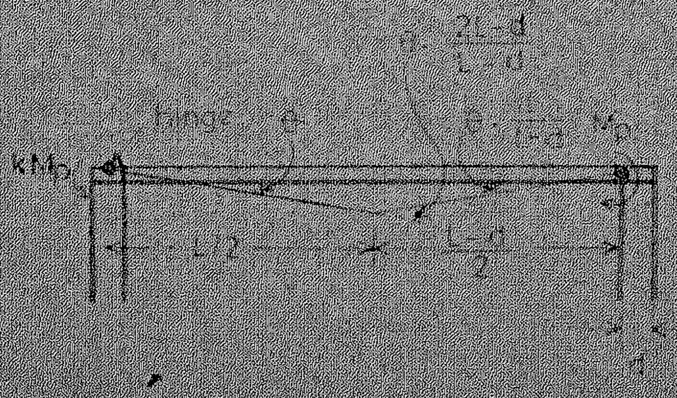


FIGURE 3.4 CORNER BEAM FAILURE CONFIGURATION

IV DESIGN OF BRACING

4.1 Aim, Function and Types

4.1.1 Aim: The bracing provision results in an efficient use of material and restricts the deformations in lateral direction to within the prescribed limits at an economic cost.

4.1.2 Function: Bracing may be designed to perform one or more of the following functions.

a. It provides stiffness to frame by carrying all the lateral loads.

b. It may share a portion of vertical loads.

4.1.3 Types: The bracing can be classified into two main types.

i) Direct bracing: This is explicitly designed to withstand the forces.

ii) Indirect bracing: This has primary utility in functional aspects of structure, yet it contributes considerably to frame stiffness. Such bracing is afforded by walls, cladding and partitions. Evaluation of this type of bracing is complicated. In the present study only direct bracing has been considered.

4.2 Assumptions

The assumptions made to achieve idealised behaviour of the frame are listed below.

a. All horizontal loads due to wind and $P - \Delta$ effects are resisted by bracing. This leads to assuming the behaviour of frame as truss.

b. Bracing is slender and offers no resistance in compression.

c. All columns in storey rotate through the same angle.

d. Axial deformations of elements are neglected.

Most of the above assumptions result in conservative design of structure. Based on the foregoing assumptions, the following relations are obtained.

i) The equilibrium of the horizontal forces is given by

$$\sum_{j=1}^{i} H_j = F_{bi} \cos\eta + \sum_{K=1}^{m+1} H_{cK} \qquad \ldots \quad (4.1)$$

where forces are as defined in Fig. 4.1.

and H_i - total storey shear at level 'i',

F_{bi} - bracing force in storey at level 'i',

H_{cK} - horizontal resistance of column 'K' in storey 'i' and

$\cos\eta = \dfrac{L_i}{L_{bi}}$.

L_i - braced bay span at level 'i' and

L_{bi} - brace length at level 'i'.

ii) The equilibrium of vertical forces is given by:

$$\sum_{j=1}^{m+1} P_{ij} = \sum_{j=1}^{m+1} F_{cij} \qquad \ldots (4.2)$$

Considering equations (4.1) and (4.2) and equilibrium of column,

$$F_{bi} = \frac{L_{bi}}{L} \left[\sum_{j=1}^{i} H_j + \theta_i \sum_{K=1}^{m+1} P_{iK} \right] \qquad \ldots (4.3)$$

where θ_i - rotation at level 'i .

4.3 Bracing Member Section

The selection of bracing section is governed by the following conditions, viz,

a. yielding of section,

b. frame stability and

c. slenderness ratio as specified by the code.

In top few storeys the slenderness ratio may govern the section and in lower storeys yielding is likely to govern the bracing section.

4.3.1 The Yielding of Bracing Section: The force in bracing member at yield is given by

$$F_{yb} = A_b * \sigma_y \qquad \ldots (4.4)$$

where F_{yb} - force in bracing member at yeild

A_b - area of bracing member

At yield - $F_{yb} = F_{bi}$ .. (4.5)

The strain in bracing is given by

$$\frac{\delta_i}{L_{bi}} = \frac{\theta_i\, h_i\, L_i}{(L_{bi})^2} \quad .. \; (4.6)$$

where δ_i - elongation in bracing member at level 'i' and

h_i - storey height at level 'i'.

The value of θ_i is obtained from equation (4.6) and Hooks law and equations (4.3) and (4.5) give,

$$A_{bi} = \frac{L_{bi}}{\sigma_y . L_i} \sum_{j=1}^{i} H_j + \frac{L_{bi}^3}{EL_i^2 h_i} \sum_{K=1}^{m+1} P_{ik} \quad .. \; (4.7)$$

4.3.1 Stability of Frame: This condition occurs only in case of gravity loads. The force in bracing is given by

$$F_{bi} = \frac{A_{bi} \;.\; E \;.\; L_i \;.\; h_i \;.\; \theta_i}{L_{bi}^2} \quad .. \; (4.8)$$

From equations (4.3) and (4.8)

$$A_{bi} = \frac{L_{bi}^3}{E.L_i^2.h_i} \sum_{j=1}^{m+1} P_{ij} \quad .. \; (4.9)$$

Maximum area obtained from equations (4.7) and (4.9) is the area of bracing section at level 'i'. This area should then be checked to satisfy slenderness ratio constraint and may have to be suitably modified.

4.4 Frame Resistance

When combined loads are acting on the frame hinge formation is as illustrated in Fig. 4.2(b), the hinge at windward end disappears as lateral deformation commences and may reappear if this deformation is allowed to continue. However, this is not acceptable, hence, at windward end the beam section is in elastic region. Using slope-deflection equations the shear resistance of a frame is given by

$$H_{Fi} = \frac{12E\,\theta_i}{h_i} \left[\sum_{j=1}^{m} \frac{K_{cj}\,K_{bj}}{K_{bj}+4K_{cj}} \right]_i \qquad . . (4.10)$$

where $K_{cj} = \dfrac{I_{cj}}{h_i}$

$K_{bi} = \dfrac{I_{bj}}{L_j}$

and I_{cj} - moment of inertia of column j at level i and

I_{bj} - moment of inertia of beam j at level i .

If frame resistance be considered then top few storeys are likely to have enough resistance with deformations restricted to within the prescribed limits and, therefore, no bracing may be required.

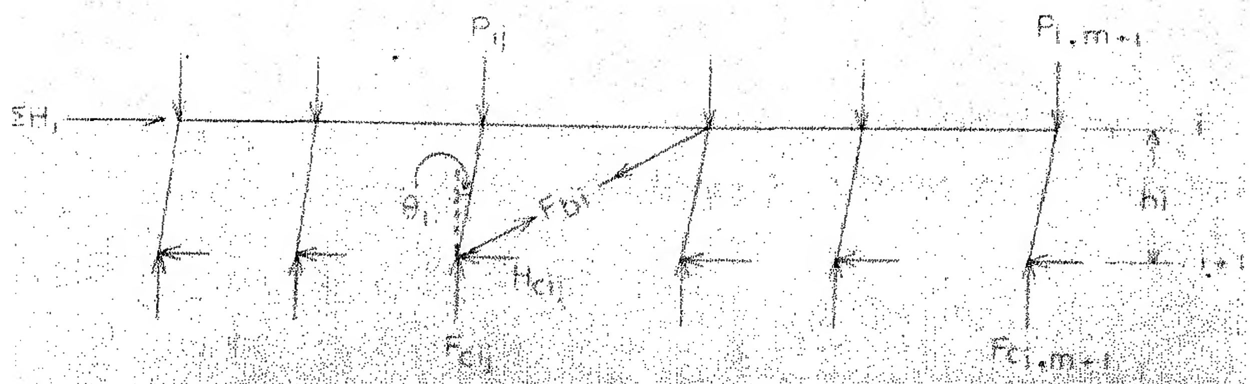
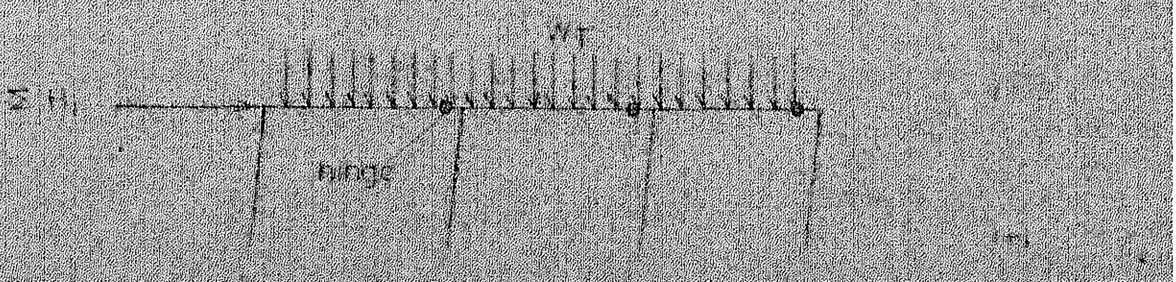


FIGURE 4.1 ONE STOREY OF A FRAME IN EQUILIBRIUM UNDER COMBINED LOADING

(a) ONLY GRAVITY FACTORED LOAD



(b) COMBINED FACTORED LOAD

FIGURE 4.2 HINGE FORMATION IN A BEAM

V FINAL DESIGN

5.1 Design of Beams

Initial design of beams was carried out at full gravity load. These sections of beams hold for final design except for braced bay beams and top corner beams.

5.1.1 Braced Bay Beams: These beams have to withstand considerable horizontal thrust at lower levels. Frame resistance for beams has been considered as given by equation (4.10). The axial thrust to be resisted by the beam at level i is given by

$$P_{bai} = \sum_{j=1}^{i} H_j + \Theta_i \sum_{K=1}^{m+1} P_{ci,K} - H_{fi} \qquad \ldots (5.1)$$

where P_{bai} - the axial thrust in the beam at level i and

H_{fi} - the frame resistance at level i.

The beam is subjected to unequal end moments. The interaction curves modified by Lehigh, are used. These curves are for slenderness ratio in neighbourhood of 40 and load ratio, $P/Py \leq 0.4$. These conditions are usually satisfied in practice. Due to unequal end moments a small error creeps in, should the curves be used directly. Therefore, the coefficient is modified as

$$W_{sij} = \left[\frac{\sigma_e - \sigma}{\sigma_e - 0.18\sigma} (1 - \sigma/\sigma_0) W_p \right]_{ij} \qquad \ldots (5.2)$$

where W_{sij} - safe load for beam at (i,j) position,

σ - axial stress in beam,

σ_e - Euler's stress,

σ_o - $\sigma_y \left[1 - \frac{\sigma_y}{4\pi^2 E} (L/r)^2 \right]$,

W_{pij} = $16\ \sigma_Y \left(\frac{S}{L_c^1} \right)_{ij}$ and

r = radius of gyration.

5.1.2 Top Corner Beam: As already discussed, the hinge may form in the column depending upon the plastic moment ratios. Thus the beam in this position may have to be redesigned. If slab resistance be considered, some economy is likely to be achieved. The Fig. 5.1 shows the bending moment diagram of composite section and Fig. 5.2 illustrates possible position of neutral axis and stress distribution. The composite section may be designed for these two cases.

Case 1: The neutral axis in the slab is as shown in Fig. 5.2(b). Equating compressive and tensile forces

$$t = \frac{A_s * \sigma_y}{0.85 * \sigma_c * b_s} \qquad .. (5.3)$$

$$M_u = T.a_1 \qquad .. (5.4)$$

where M_u - plastic moment of composite section,

t - depth of slab in compression,

σ_c - cube strength,

T	-	tensile force,
b_s	-	slab width acting in conjunction with beam given by code,
a_1	-	lever arm and
A_s	-	area of steel.

Case 2: The neutral axis in steel section is as shown in Fig. 5.2(c). The plastic moment is given by

$$M_u = c_1 * a_1 + c_2 * a_2 \qquad \ldots (5.5)$$

where

c_1	-	compressive force of concrete slab,
c_2	-	compressive force of steel portion and
a_1, a_2	-	lever arms for c_1 and c_2 respectively.

The total moment capacity is

$$M_u + M_p = \frac{C_1 * W_T * L_c^2}{8} \qquad \ldots (5.6)$$

5.2 Column Design

The design of frame column is comparatively complicated. The design is based on discretising the system through subassemblages and utilising $M - \phi$ relationship to ascertain the maximum resistance of column at given rotation.

5.2.1 Loading Conditions: For final check column needs to be designed for,

a. gravity loading and

b. combined gravity and wind loading.

5.2.2 Gravity Loading: Further checks have to be applied when,

a. all the spans are fully loaded and

b. alternate spans are fully loaded.

5.2.3 Design Classification: The column design can be classified based on

a. the loading condition,

b. the position along vertical axis, viz, roof column or floor column,

c. the position along horizontal axis, viz, exterior column or interior column,

d. the adjacent span ratio, viz, equal spans or unequal spans,

e. number of bays and

f. the status of bay, viz, braced or unbraced bay.

5.2.4 Gravity Load Case (all spans loaded):

a. Outer Columns: The Fig. 5.3 shows a fully loaded frame. The moments and shear forces acting on the face of the column as illustrated in Fig. 5.3(b) are given by equation (3.8). The Fig. 5.4(b) shows the effect on moment capacity, which is negligible for $P/P_y \leq 0.6$ and $h/r \leq 40$ and for fairly large variation of C. Since C

does not have appreciable effect, its value for exterior column is assumed as unity. Then the moment

$$M_{cij} + M_{ci-1,j} = M_{ij} \quad \ldots \quad (5.7)$$

b. Interior Columns: The interior columns may have equal adjacent spans or unequal adjacent spans.

Case 1: Equal Adjacent Spans: The Fig. 5.3 shows forces acting on column face, since

$$L_{c\,j-1} = L_{cj} \quad \ldots \quad (5.8)$$

$$M_{ijR} = M_{ijL} \quad \ldots \quad (5.9)$$

where M_{ijR} - moment at (i,j) position on right face of column and

M_{ijL} - moment at (i,j) position on left face of column.

The column, therefore, does not undergo any rotation deformation and is designed as axially loaded column based on Column Research Council, (CRC), formula. The critical load capacity, of column, is given by

$$P_{cr} = P \left(1 - \frac{\sigma_y}{4\pi^2 EI} \cdot K_e h^2\right) \quad (5.10)$$

where K_e - effective length factor.

Case 2: Unequal Adjacent Spans: In Fig. 5.3(c) if $M_{ijR} \neq M_{ijL}$, then there acts some moment on column ends. These moments have the same sense and, therefore, the design procedure is similar to that of exterior column.

5.2.5 Gravity Load Case (Alternate spans loaded): The alternate spans loading causes change in ratio of end moments acting on the column and for top few storeys, there is significant change in P/P_y ratio.

a. Exterior Columns: For a fully loaded frame except for the span (i-1,j), shown in Fig. 5.5(a), the change in moment at joint of interest, at centre line, as shown in Fig. 5.5(b) is

$$\delta M_{ij} = M_{pbi,j} - (M_D + W_D * \frac{d}{2} * L_c)_{ij}$$

$$+ (W_T * L_c * \frac{d}{2})_{ij} \qquad \ldots \quad (5.11)$$

where δM_{ij} - change in moment at joint (i,j),

M_{Dij} - fixed end moment due to dead load at joint (i,j) and

W_D - factored dead load/unit length.

The condition

$$0 < M_{Dij} < M_{pbij} \qquad \ldots \quad (5.12)$$

must be satisfied which results in

$$\frac{W_D}{W_T} \leq \frac{3}{4} \quad \quad (5.13)$$

All the parameters for the column are now determined and column section can be checked. For value of $P/P_y \geqslant 0.6$ the variation of C, i.e., the moment ratio causes appreciable change in moment resisting capacity as shown in Fig. 5.6.

b. Interior Columns:

Case 1: Equal Spans Two Bays: Consider the frame as shown in Fig. 5.7(a). The moment resulting from formation of beam mechanism in bay (i,2) is resisted by columns (i-1,2) and (i,2) and beam (i,1). The subassemblage is as shown in Fig. 5.8. The end of the beam at the joint (i,1) is assumed to be hinged. The moment acting at joint position is

$$M_{ij} = (K_s M_{pb} + \frac{V \times d}{2})\ i,2 \quad \quad (5.14)$$

where K_s - strain hardening factor and

V - shear force acting on the face of the column when W_T is load on beam.

The magnitude of rotation for hinge to be formed in beam is given by

$$\phi_{pb\ i,j-1} = \frac{2}{3} * f * \frac{\sigma_y}{E} \left[\left(1 - \frac{4}{3} \frac{W_D}{W_T}\right) * \frac{L_c}{d_b} \right]_{i,j-1} \quad \cdot\cdot \quad (5.15)$$

where ϕ_{pb} - rotation at hinge formation

f - shape factor, 1.12.

The maximum moment occurs at ϕ_{pb} and maximum moment resisting capacity of the joint can be obtained as illustrated by Fig. 5.9.

Case 2: <u>Two Adjacent Spans, Three Bays</u>: If another span is added to the left of Fig. 5.7(a), the idealised behaviour of subassemblage comprising the same member for joint (i,j) undergoes minor change as shown in Fig. 5.10. The joint (i,j-1) is subjected to rotation and the beam deflects as shown. It is assumed that $\delta M_{ij} = \delta M_{i,j-1}$ and, therefore, rotations are equal. The plastic hinge rotation is given by

$$\phi_{pb\ i,j-1} = f \frac{\sigma_y}{E} \left[\left(1 - \frac{4}{3} \frac{W_D}{W_T}\right) \frac{L_c}{d_b} \right]_{i,j-1} \quad \cdot\cdot \quad (5.16)$$

Case 3: <u>Unequal Spans, Two Bays</u>: The subassemblage is same as shown in Fig. 5.8. However, the column may either have a half wave elastic curve when 'C' is approximately equal to -1, or it may have an elastic curve of two unequal reversed half waves. The frame is as shown in Fig. 5.11. Assuming that loading is same per unit area, the maximum unbalanced moment at joint (i,2) occurs when span (i,1) is fully loaded and span (i,2) has dead load only, as shown. The moment relation is given when,

$$(M_{pb} + \frac{V*d}{2})_{i,1} > (M_{pb} + \frac{V*d}{2})_{i,2} \quad \ldots \quad (5.17)$$

Considering the joint (i-1,2)

$$M_{i-1,2} = (\frac{W_D * L_c^2}{12} + \frac{V*d}{2})_{i-1,1} - (M_{pb} + \frac{V*d}{2})_{i-1,2} \quad \ldots \quad (5.18)$$

where $M_{i-1,2}$ - moment acting at joint (i-1,2)

(i) If $M_{i-1,2} = 0$ .. (5.19)

then $C \simeq 0$

(ii) If $M_{i-1,2} > 0$ .. (5.20)

then column bends in two unsymmetrical half waves. In this case $C \simeq 0$ is assumed, this being conservative approximation.

(iii) If $M_{i-1,2} < 0$ .. (5.21)

then the column bends as a single half wave and $C = -1$, this is about the worst case, that can occur for a column and is again a conservative approximation.

The imposed moment on joint of interest having been known $\Phi_{pbi,2}$ can be obtained as given by equation (5.15) and column sections can be checked.

Case 4: Unequal Spans, Three or more Bays: In this case the subassemblage is same as shown in Fig. 5.10 and equation (5.16) may be used.

5.3 Design of Column for Wind and Gravity Loads:

Due to truss action, as assumed (refer Fig. 4.1), the unloading occurs on wind ward column and additional load is imposed on leeward column. The safety of previously selected column should be checked by following equation.

$$\delta P_{ci,j} = \sum_{k=1}^{i} H_{k} * \tan\eta + \delta P_{ci-1,j} \qquad \cdots \quad (5.22)$$

where $\delta P_{ci,j}$ - additional column load due to wind

and

$$P_{ci,j} = -\delta P_{ci,j-1} \qquad \cdots \quad (5.23)$$

The additional column load should be within the capacity of the section provided. Thus all the columns for any given load conditions and any type of rectangular, regular frame can be designed.

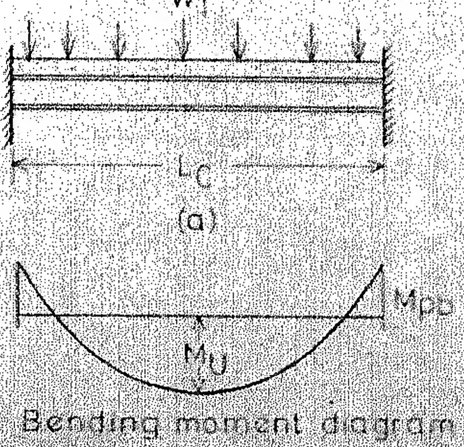


FIGURE 5.1 COMPOSITE BEAM ACTION

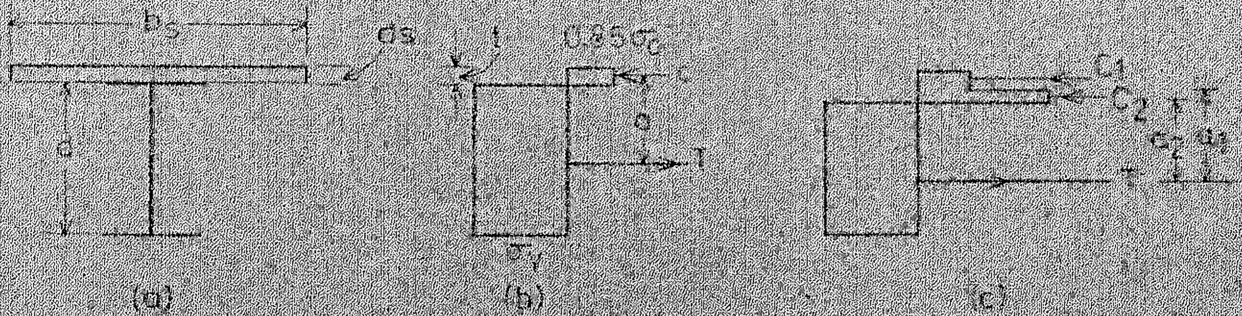


FIGURE 5.2 EQUILIBRIUM OF FORCES AT YEILD IN COMPOSITE SECTION

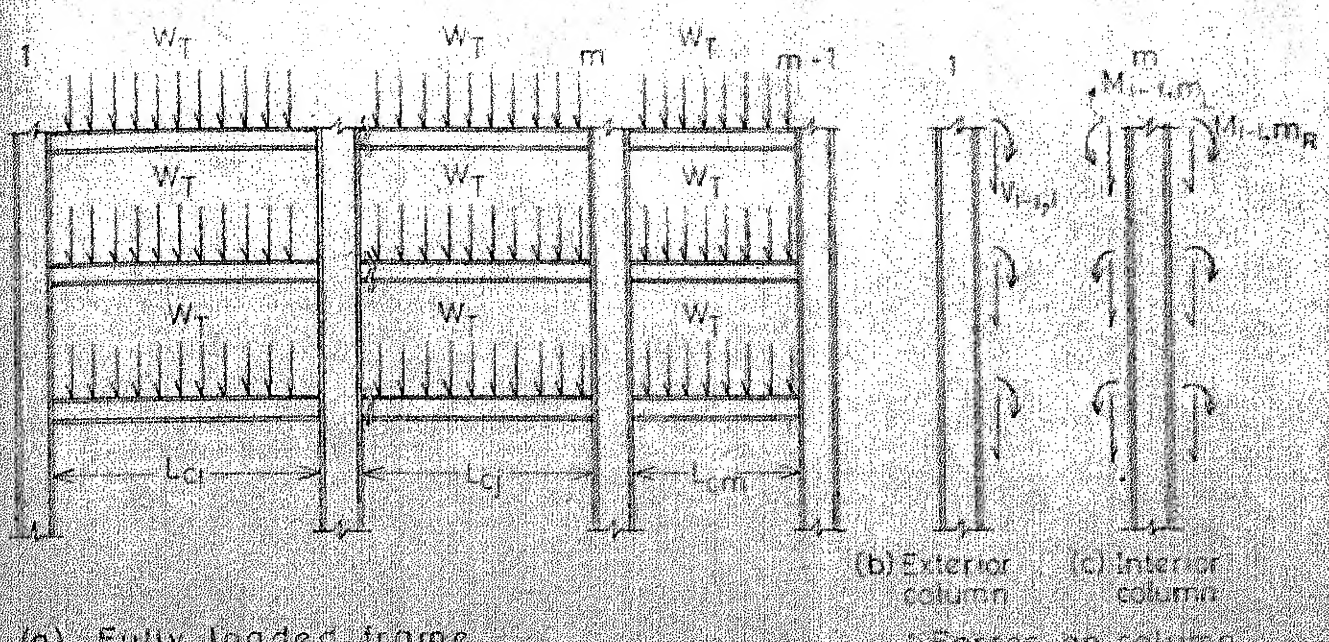


FIGURE 5.3 GRAVITY CASE – ALL SPANS FULLY LOADED

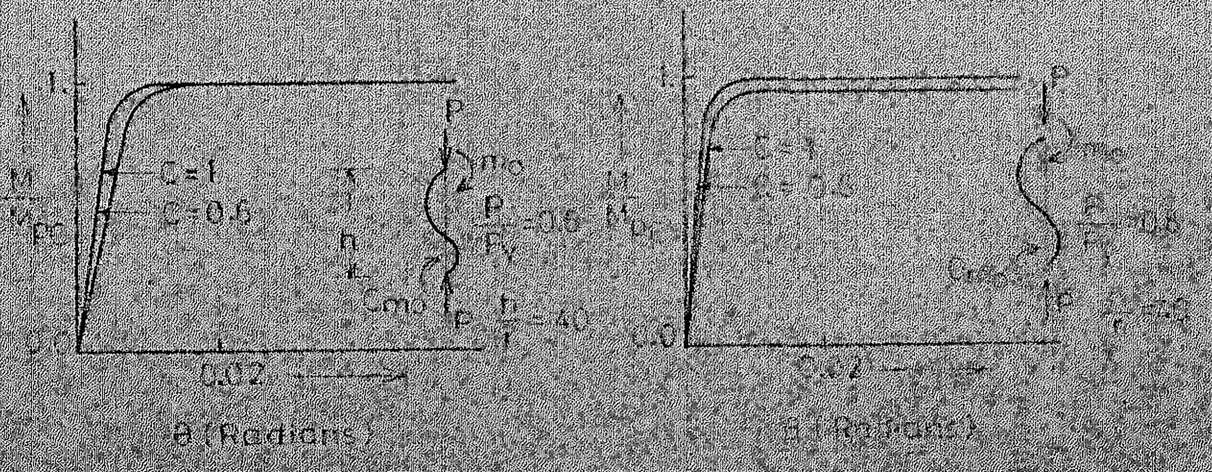
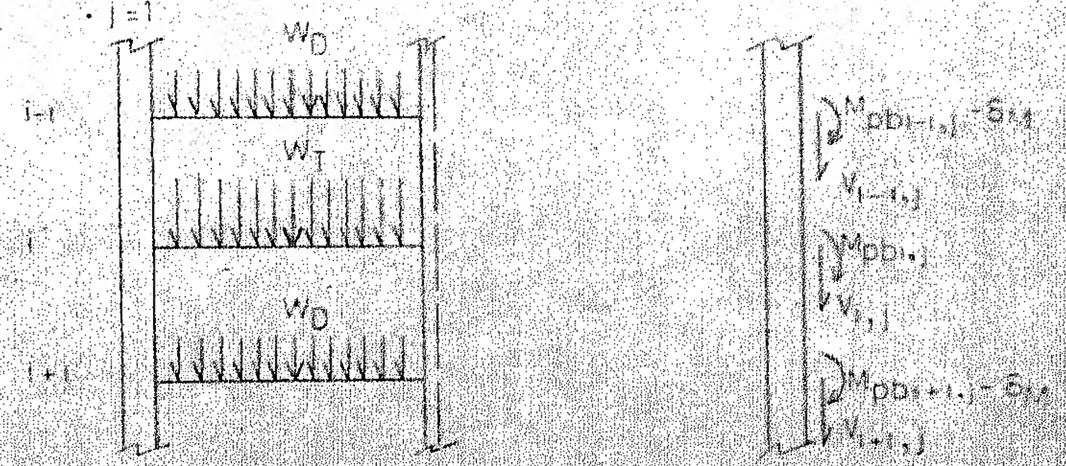


FIGURE 5.4 MOMENT ROTATION COMPARISON



a) Exterior column case b) Moments on column

FIGURE 5.5 EXTERIOR COLUMN-ALTERNATE SPANS LOADED

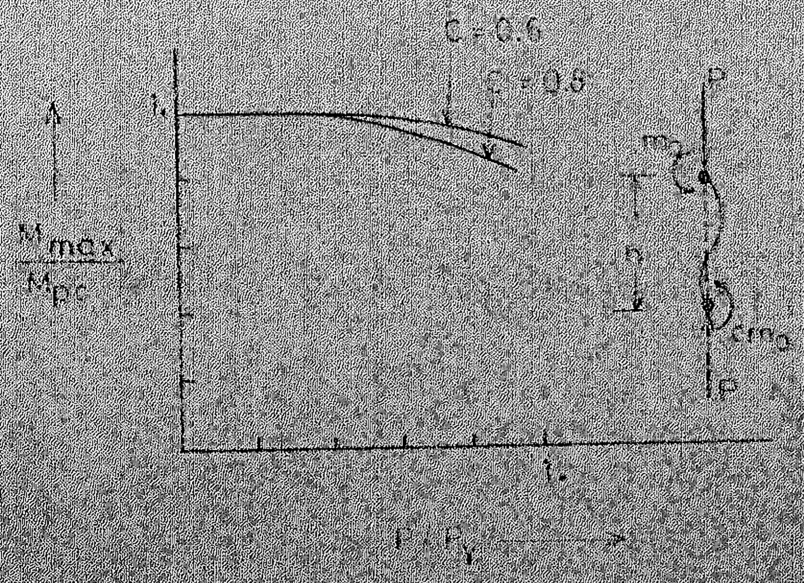


FIGURE 5.6 MOMENT-RATIO — LOAD RATIO CURVE

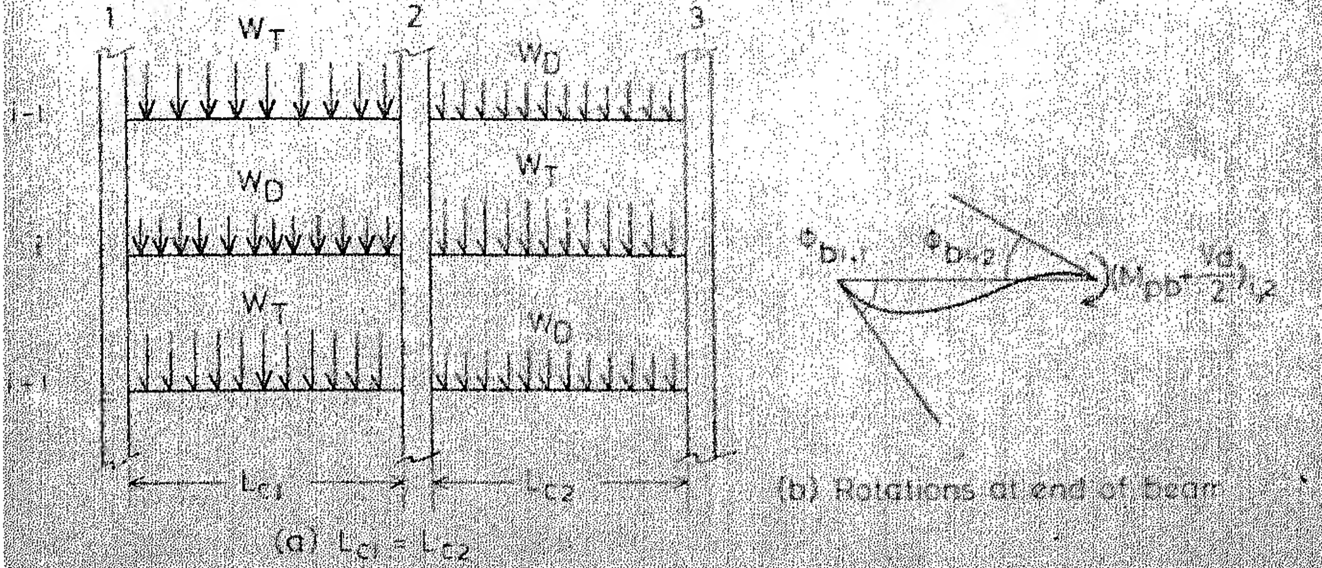


FIGURE 5.7 ALTERNATE SPANS LOADED (EQUAL SPANS)

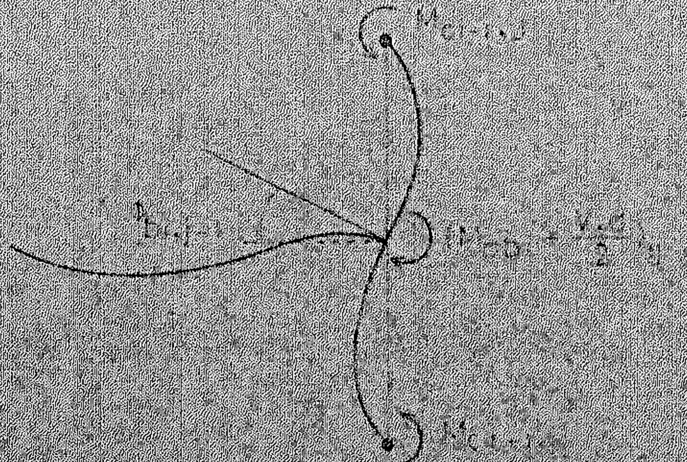


FIGURE 5.8 SUB ASSEMBLAGE MECHANISM (EQUAL SPAN)

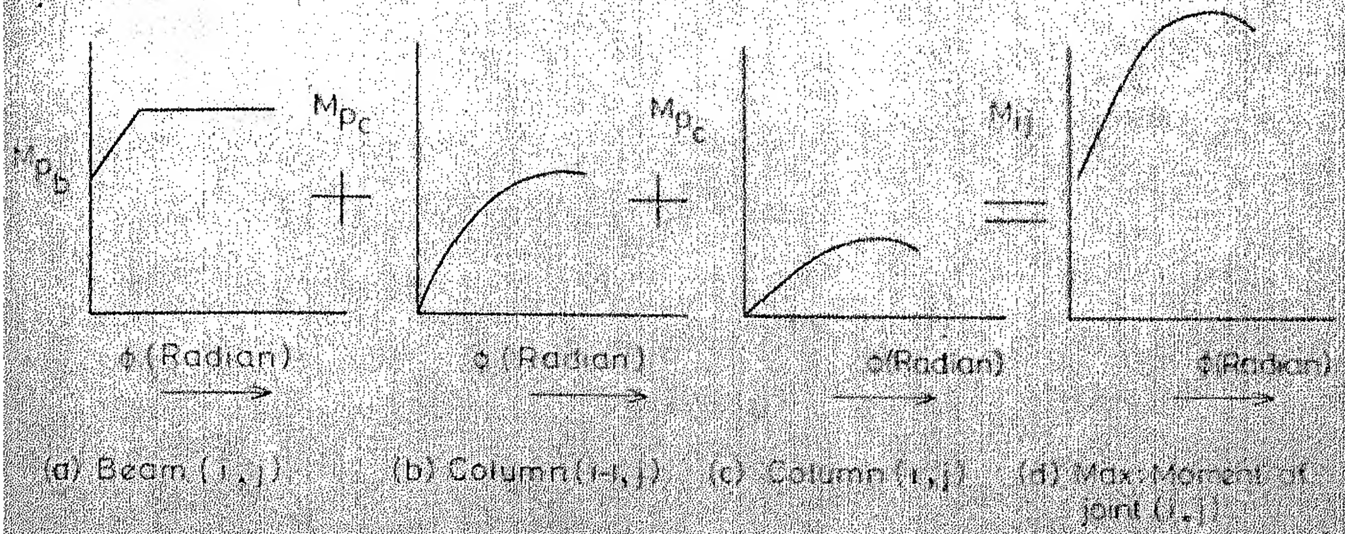


FIGURE 5.9 JOINT MOMENT CAPACITY COMPUTATION

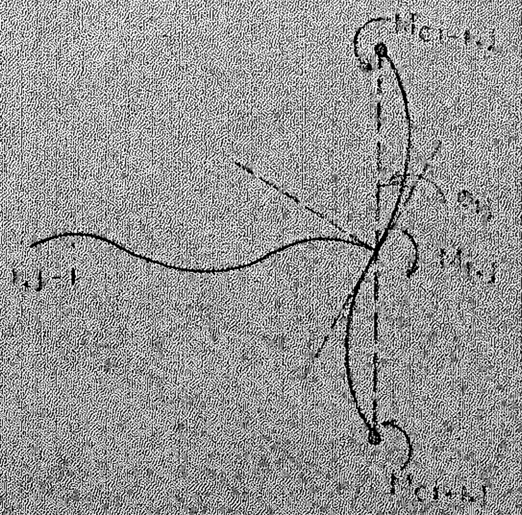


FIGURE 5.10 SUBASSEMBLAGE FOR MORE THAN 2 SPANS

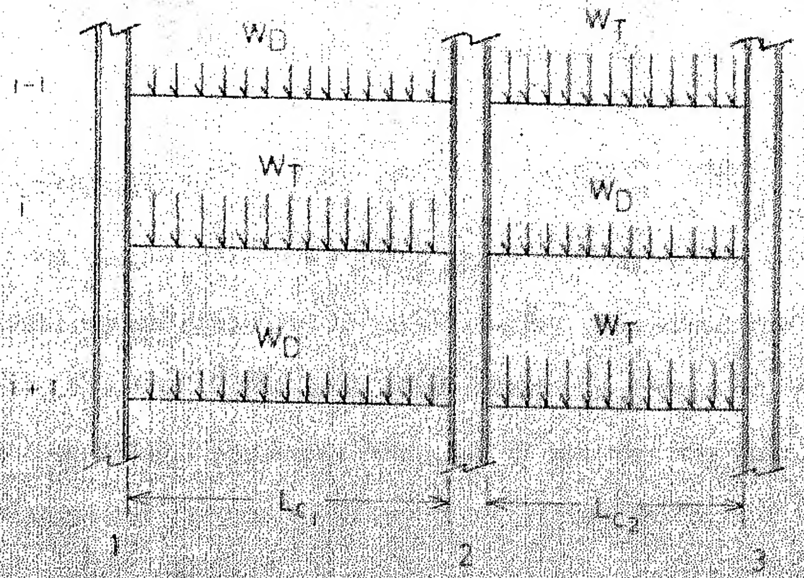


FIGURE 5.11 ALTERNATE SPANS LOADED(UNEQUAL SPANS)

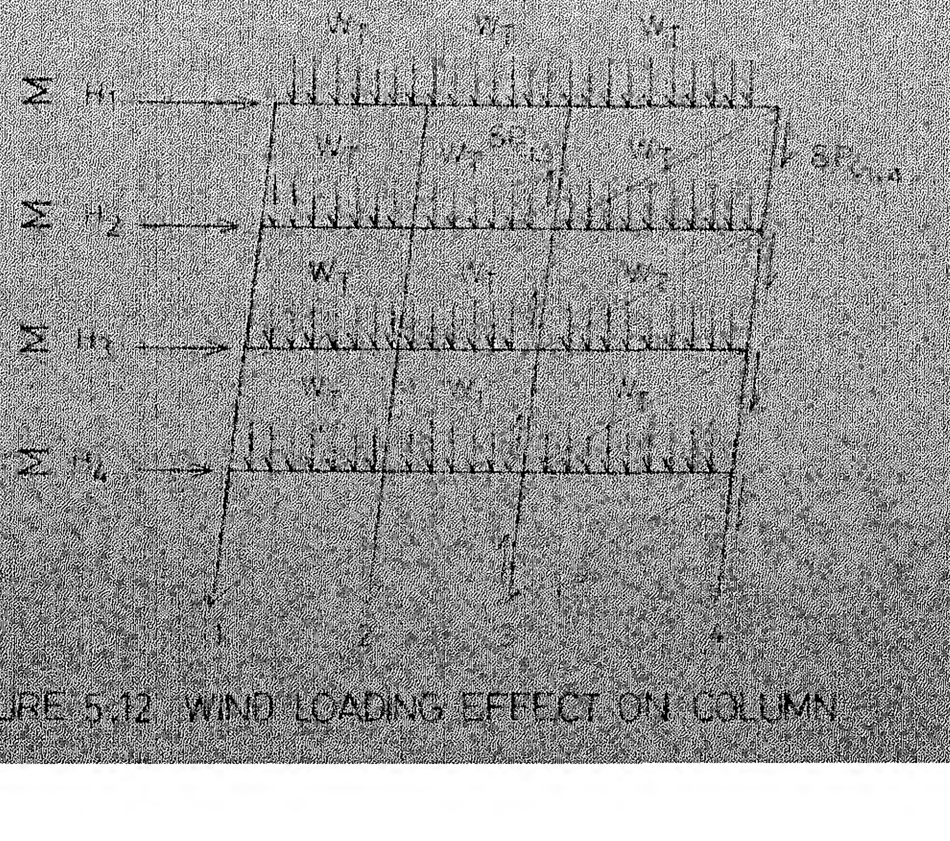


FIGURE 5.12 WIND LOADING EFFECT ON COLUMN

CHAPTER VI

COMPARISONS OF RESULTS AND CONCLUSIONS

6.1 General

A comprehensive computer programme was developed for the design of multistorey multibay rectangular frames. The programme was tested for correctness by feeding in the data of the frame as presented by Lehigh (1965). The effect on weight and hence the cost of a three bay frame was studied by incorporating various modifications, listed as follows:

a) Higher ratio of column load to yield load.

b) Frame resistance.

c) Composite beam action.

d) Different adjacent span ratio for a given total span.

c) Spacing of frames.

d) Choice of braced bay.

Also a four bay frame has been studied for various adjacent span ratios.

The reference frame being Lehigh (1965), the FPS system has been used in this thesis. The programme can be readily modified for any other system.

6.2 The Design Feature

These features of the reference Lehigh (1965) frame are,

i) load factor	-	gravity case	1.7
		gravity + wind case	1.3
ii) dead load	-	roof	95 lbs/sft
	-	floor	120 lbs/sft
iii) live load	-	roof	30 lbs/sft
		floor	100 lbs/sft
		wind	20 psf
iv) storey height			12.0 ft
v) number of storeys			24
vi) number of bays			3 (20'-12'-28')
vii) spacing of frames			24.0 ft
iix) yield stress of steel			36 ksi

The results of the test frame are presented in Table 6.1. The saving in the overall weight is due to,

i) higher ratios of column load to yield load,

ii) selection of a column section for each storey unlike in the reference solution where in the column sections remain same for two storeys,

iii) the iterative process in preliminary selection of sections such that these are just sufficient for short columns and are revised in subsequent checks,

iv) a lower average weight (325 lbs/rft) of the column than the reference frame (625 lbs/rft) and

v) marginal saving in weight occuring from consideration of beam stiffness while evaluating the overall frame resistance.

6.3 Frame Resistance

The frame resistance is given by Wright (1972) as

$$H_{fi} = (M_{ci,1b} + M_{ci,1t} + 2 \sum_{j=2}^{m+1} M_{pci,j} - \theta_i h_i \sum_{j=1}^{m+1} P_{i,j})/h_i \quad (6.1)$$

where $M_{ci,1b}$ - elastic moment of column at (i,1) at lower end,

$M_{ci,1t}$ - elastic moment of column at (i,1) at top end,

$M_{pci,j}$ - plastic moment of column at (i,j) and

If $H_{fi} > \sum H_i$ . . . (6.2)

No bracing is required for storey 'i'.

This approach does not seem to be correct and needs to be modified as follows

$$H_{fi} * h_i \geq \sum_{j=1}^{i} H_j \left(\sum_{k=j}^{i} h_k \right) \quad . . . (6.3)$$

In the reference solution the frame resistance has not been considered while designing the bracing. As will be seen from Table 6.1 the weight of bracing remains constant over top few storeys, which is due to the slenderness ratio governing the bracing design. The column has additional load capacity due to wind and gravity load combination because of lower load

factor, this additional capacity can be readily utilised to resist horizontal shear thus leading to redundancy of bracing in top few stories as is reflected by the results in Table 6.2. The percentage saving is nominal for tall frames but considerable in the case of frames upto ten storeys. However it is essential to check for deflection of the frame by elastic approach.

6.4 Composite Beam Slab Action

The results for the reference frame taking into account the composite beam slab action are as given in Table 6.3. The saving in beam sections of shorter spans are likely to be offset by the revision of column sections due to net increment on column ends which occurs particularly when adjacent spans have large differences.

6.5 Span Variations

The frame under study is three bay, ten storey frame. The weights of frames for various span ratio configurations have been evaluated and presented in Table 6.4. The following are the observations:

i) The weight is minimum when all spans are equal or nearly equal.

ii) The weight is likely to be more for unsymetric frames.

iii) The gravity case is likely to govern column sections for less number of storeys and lower wind pressures.

v) Occasionally it is possible to get a heavier column section in upper storey as compared to lower one, this is for a simple reason that the reserve strength of the joint is combination of upper and lower columns resisting the beam moment.

6.6 Spacing of Frames

The frame considered is ten storey, three bay (20'-12'-28') frame. The effect of spacing is studied by varying the dead load at the rate of 10 lbs/3 ft change in spacing of frames. The results are shown in Table 6.5. The results indicate that the saving in over all weight of frame is marginal due to coarse discrete section range availability.

6.7 Choice of Braced Bay

The frame considered for this parameter is ten storey, three bay (20'-12'-28') frame. The choice of braced bay has some effect on the over all weight as shown in Table 6.6, from which the following conclusions can be drawn.

i) The angle of bracing effects the column section profoundly. The revision for (10,3) column when braced bay is 28.0 ft, takes place from 158 to 167 lbs/rft, and for the same column for 12.0 ft braced bay span the revision is from 158 to 176 lbs/rft.

ii) The weight for 12.0 ft braced bay is lesser than 28.0 ft braced bay. However, for very tall structures this may not be valid, as the advantage gained in reduced bracing weight will be offset by sharper increase in column section revisions due to increased axial loads occuring from wind load.

iii) The braced bay span of 20.0 ft yields minimum weight because of no revision of sections due to wind load.

6.8 Four Bay Frame

The results of four, four bay frames are presented in Table 6.7. It will be readily seen that the maximum weight for any span configuration of this case is considerably less than the lowest obtained for any three bay frame (Table 6.4). This is due to the lesser unbalanced moments imposed on column ends.

6.9 General Conclusion

From the foregoing study carried out on various frames the following conclusions can be drawn.

i) The adjacent span ratio of bays has a profound effect on the overall weight of structure.

ii) The minimum weight of frame is obtained for equal or nearly equal bays.

iii) Consideration of frame resistance eliminates bracing from the top few storeys resulting in economy.

iv) The spacing of frame does not have much effect on the overall weight of structure.

v) The composite beam slab action may reduce the beam sections and may necessitate heavier column sections.

vi) Staggering of the braced bays will eliminate revision of column sections.

vii) The angle of bracing has profound effect on taller structures than others.

6.10 Future Study

Further study should be conducted taking following points into account:

i) Incorporation of stiffness of for [illegible] [illegible] joints in sub-assemblage behaviour.

ii) Beam column action of composite sections.

iii) Reduction or total elimination of secondary net work of beams and evolving a flooring system acting in unison with the frame.

TABLE 6.1 WEIGHT OF 24 STOREY COMPARISON FRAME

	Weight of beam for Storey (Lbs)	Weight of Column for Storey (Lbs)	Weight of Bracing Members for Storey (Lbs)	Total Weight upto Storey (Lbs)
1	1830	2736	335	4901
2	2372	2220	334	9827
3	2360	3132	334	15653
4	2333	3888	334	22208
5	2333	4416	334	29291
6	2333	4680	334	36638
7	2333	5820	380	45171
8	2333	5928	455	53886
9	2333	6564	455	63238
10	2333	7056	501	73128
11	2333	7656	565	83782
12	2333	8232	709	94955
13	2333	8856	709	106852
14	2333	9372	824	119381
15	2333	9900	824	132438
16	2333	10644	824	146238
17	2333	11184	934	160689
18	2333	11880	934	175835
19	2333	12360	1043	191570
20	2333	13352	1043	208278
21	2333	13440	1130	225180
22	2333	14544	1262	243319
23	2333	15432	1262	262346
24	2333	16440	1262	282380
	Weight by Lehigh solution			296400
	Weight by elastic solution (Lehigh)			316000

TABLE 6.2 WEIGHT OF 24 STOREY FRAME INCORPORATING FRAME RESISTANCE

	Weight of beam for Storey (Lbs)	Weight of Column for Storey (Lbs)	Weight of Bracing Members for Storey (Lbs)	Total Weight upto Storey (Lbs)
1	1830	2736	0	4566
2	2372	2220	0	9158
3	2360	3132	0	14650
4	2333	3888	0	20870
5	2333	4416	334	27953
6	2333	4680	334	35300
7	2333	5820	380	43833
8	2333	5928	455	52549
9	2333	6564	455	61901
10	2333	7056	501	71791
11	2333	7556	565	82344
12	2333	8232	709	93617
13	2333	8856	709	105515
14	2333	9372	824	118043
15	2333	9900	824	131100
16	2333	10644	824	144901
17	2333	11184	934	159351
18	2333	11880	934	174497
19	2333	12360	1043	190233
20	2333	13332	1043	206940
21	2333	13440	1130	223843
22	2333	14544	1262	241981
23	2333	15432	1262	261008
24	2333	16440	1262	281043

TABLE 6.3 COMPOSITE BEAM ACTION 24 STOREY FRAME

	Weight of beam for Storey (Lbs)	Weight of Column for Storey (Lbs)	Weight of Bracing Members for Storey (Lbs)	Total Weight up to Storey (Lbs)
1	1760	2652	335	4747
2	2341	2232	334	9655
3	2333	3264	334	15586
4	2283	3888	334	22091
5	2283	4416	334	29124
6	2283	5004	334	36745
7	2283	5928	380	45336
8	2283	5928	455	54004
9	2283	6660	455	63400
10	2283	7164	501	73348
11	2283	7656	565	83582
12	2283	8340	709	95183
13	2283	8856	709	107031
14	2283	9480	824	119618
15	2283	9996	824	132720
16	2283	10644	824	146471
17	2283	11184	934	160872
18	2283	11880	934	175968
19	2283	12360	1043	191654
20	2283	13332	1043	208312
21	2283	13440	1130	225164
22	2283	14820	1262	243529
23	2283	15432	1262	262505
24	2283	16440	1262	282490

TABLE 6.4 WEIGHT OF 10 STOREY FRAME - SPAN VARIATION

Span inches	240-144-336	180-360-180	192-336-192	204-312-204	216-288-216	228-264-228	240-240-240
Storey	Weight	Weight	Weight	Weight	Weight	Weight	Weight
1	4900	4519	[illegible]	3907	3727	3715	[illegible]
2	9827	9242	8853	8170	7937	7717	[illegible]
3	15652	14707	14071	12829	[illegible]	12373	[illegible]
4	22207	[illegible]	19714	18039	[illegible]	[illegible]	17259
5	29290	27088	26386	[illegible]	[illegible]	23365	23192
6	36637	34119	33142	30775	[illegible]	30225	30157
7	45170	41729	40696	[illegible]	38513	37766	37805
8	58834	50674	49272	45976	46570	45708	45857
9	63186	60058	58362	[illegible]	55033	54047	54350
10	72968	69879	67912	[illegible]	64099	62996	63109

TABLE 6.5 WEIGHT OF 10 STOREY FRAME - SPACING VARIATION

Frame spacing inches	180	216	252	288	324
Storey	Weight	Weight	Weight	Weight	Weight
1	3730	3969	4562	4900	5084
2	7164	7990	8865	9827	10534
3	11031	12498	13953	15652	17154
4	14929	17172	19174	22207	24278
5	19257	22357	25294	29290	31714
6	23825	27902	31966	36637	40363
7	28741	34035	29203	45170	49332
8	33993	40564	46977	53834	59037
9	39810	47451	55171	63186	69584
10	46022	55082	63596	72968	80711
Weight for 10 ft spacing	30681	30601	30284	30403	29893

TABLE 6.6 WEIGHT OF 10 STOREY FRAME - BRACED BAY VARIATION

24 FT SPACING

Span inches	240-144-336	240-144-336	240-144-336
Braced span	336	144	240
Storey	Weight	Weight	Weight
1	4900	4651	[illegible]
2	9827	9320	[illegible]
3	15652	14914	150[illegible]3
4	22207	21274	21[illegible]46
5	29290	28189	2839[illegible]
6	36637	35406	35636
7	45170	43802	4407[illegible]
8	53834	52427	52677
9	63186	61818	61929
10	72986	71748	71633

TABLE 6.7 WEIGHT OF 10 STOREY FRAME - FOUR SPAN CONFIGURATION

24 FT SPACING

Span inches	120-240-240-120	132-228-228-132	144-216-216-144	156-204-204-156	168-192-192-168	180-180-180-180
Storey	Weight	Weight	Weight	Weight	Weight	Weight
1	3172	3246	2980	2871	2945	2838
2	6962	6805	6564	6249	6294	6236
3	11557	10872	10710	10206	10215	10210
4	16457	15489	15433	14581	14641	14677
5	21999	20832	20497	19627	19661	19758
6	28364	26883	26425	25298	25506	25475
7	35676	33568	33070	31803	32209	31725
8	43365	41157	40522	39211	39814	39848
9	51441	49227	48442	47542	48100	48195
10	60056	57691	57312	56299	56896	56913

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